

Advances in the Use of Structural Glass

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This paper focuses on three key areas in the design and use of glass as a structural material, illustrating these areas with the use of Apple Store Upper West Side as a case study: 1. Introduction to the structural principles required to design transparent structures; 2. The design of large glass structures, from concept through design development to final details, from global structural models to local stresses in connections; 3. The variations in structural capacity of glass with changes in load duration, methods of load application and location of high stresses.

Keywords: Jumbo, Cold Bent, Transparency, Structural, Finite Element Analysis

1. Introduction

The Author notes that while there are many discussions on the material properties of glass, and equally many discussions on structural principles in general, little is written on the total route of design of glass structures from concept to completion. The following text outlines the work we at Eckersley O'Callaghan (EOC) have completed over the last two years, with an attempt to show the process through which we go to ensure large scale glass structures function. It is not intended to be a definitive method of structural glass design, merely an introduction into the way we at EOC approach and carry out our work. By way of example, the paper follows the development of the most recent “significant” and structurally complex Apple Store, Upper West Side, New York, NY.



Figure 1: Apple Store, Upper West Side, front elevation

This paper is written primarily for three audience groups: The first is the Structural Engineer who works with glass, to outline the advances in structural glass design in recent years and to demonstrate how the state of the art is progressing; The second is the Structural Engineer who has underlying knowledge of structural systems but unsure of how to apply them to the design of glass structures; The third is the Architect and Designer who would like to gain a deeper understanding of the work a Structural Engineer carries out to realise the architectural vision - in an effort to bring the latter group closer to the world of the Structural Engineer, it is hoped that some insight might be gained in the challenges of making modern minimal glass structures stand up.

2. Apple Store Upper West Side

The building is flanked by two 15m tall vertical stone-clad walls along the perimeter which support the glass roof and glass wedge. These two distinct parts are vertically supported by 5 cable trusses, spaced 5m apart (Figure 2, and Figure 3), which span 20m between the flanking stone walls. The main barrel vault roof is made up of curved insulated glass panels (Figure 3, Figure 4). While a challenge in themselves, the trusses and curved IGUs (2.3m x 4.6m) are beyond the scope of this paper.

The 13m wedge-shaped glass façade runs along Broadway and 67th St. (Figure 6, Figure 7) which is capped by a cold-bent laminate-spliced glass roof (Figure 5). This paper focuses on the structural principles and design methods used to design the glass wedge, with particular focus on the cold-bent splice-laminated load-transferring roof.

3. The Structural Concept and Principles

As with all buildings, the structural concept begins with an architectural brief which is developed to satisfy basic structural load cases: vertical (gravity, snow, wind) and lateral loads (wind, seismic forces) in all plan directions. For light-weight and “open” structures such as Apple UWS, wind load is often governing with a particular need to transfer lateral loads through bracing systems into the ground.

In an effort to increase the transparency of a structure, the aim is to transfer all loads through glass and minimal glass connections into the ground or perimeter walls. The use of steel was limited to the 20m trusses and purlins running from the rear to the front of the store: glass does not yet lend itself to 20m spans, and steel has the benefit of being able to hide services, such as sprinklers and cabling running through the purlins.

To accommodate and transfer all loads to the ground and support system in the stone walls, the wedge is divided into 6 structural zones (Figure 8), each of which combine to form local structural systems (Figure 9, Figure 10, Figure 11). Zones A, B, C and D take lateral loads and transfer them to the roof and ground, as well as bracing the structure in lateral directions. Zones E and F brace zones A, B, C and D against wind and seismic loads. Zones A and D also brace each other. These structural zones were developed with close collaboration within the design and construction team – it is the job of the Structural Engineer to ensure what is proposed can be built in a way that is acceptable to the Architect and Client with careful consideration of aesthetic requirements, geometry effects, material limitations, technical possibilities and the wish to push the envelope of what is possible with structural glass.

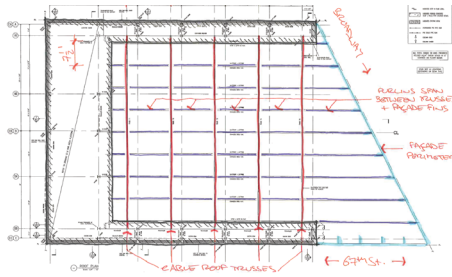


Figure 2: Structural plan. Steel trusses span 20m between the stone perimeter walls. Broadway runs diagonally from SE to NW along a 25m facade, 67th St runs horizontally from E to W along a 12m facade. Steel purlins run between 22-33m from the back to the front of the store with a movement joint at the wedge/barrel vault interface.

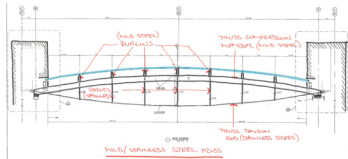


Figure 3: Two parallel mild steel plates, form the top chord of the truss. A single pre-stressed 38.1mm high strength "nitronic" stainless steel rod ($\sigma_y = 1\text{GPa}$) forms the bottom tension chord.

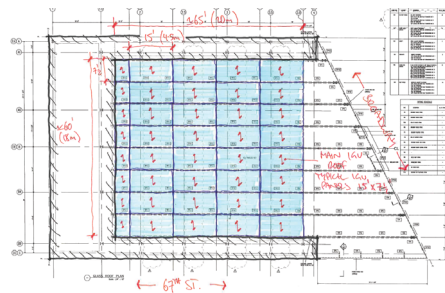


Figure 4: Curved IGU roof plan: 4.6m x 2.3m curved IGU panels, 10kN each. The curved geometry was developed to balance self-weight deflections to form a continuous radius over the barrel vault.

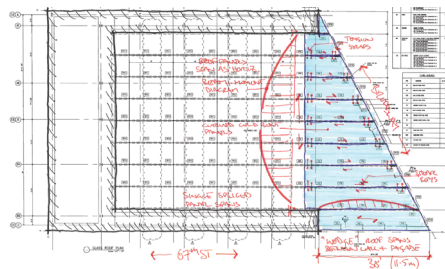


Figure 5: Wedge roof plan. The middle 8 panels are bolted together to act as a horizontal beam spanning between the stone walls on the perimeter.

The single panel on the bottom of the wedge acts as horizontal beam spanning between the end of the stone wall and façade.

All panels could bent and laminate-spliced.

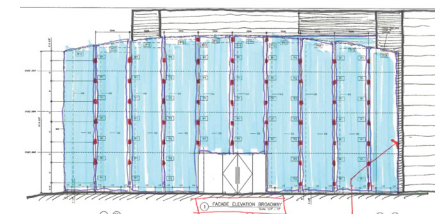


Figure 6: Broadway façade elevation. Adjacent panels are spliced to form three groups of glazing which cantilever from bolted connections into the ground to resist in-plane loads and to stabilise the wedge roof.

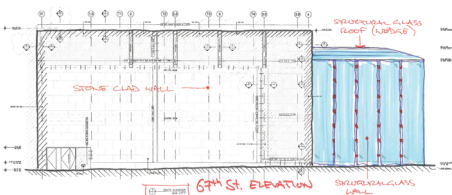


Figure 7: 67th St. elevation. The 5 panels on this elevation are spliced together to act as a single shear wall resting in-plane walls.

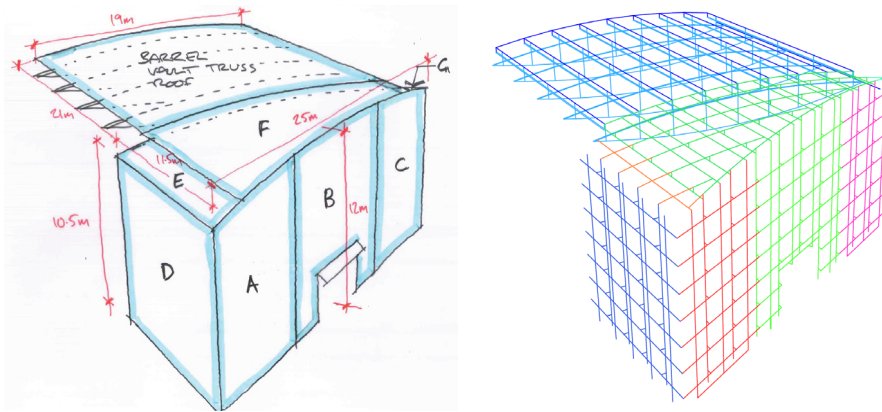


Figure 8: Structural zones within the wedge. **Zone A:** 3No. adjacent full height splice-laminated glass panels mechanically stitched together form a vertical cantilever in the facade plane; full height fins span from ground to purlins in zone F; bolted to ground slab; **Zone B:** 4No. adjacent full height splice-laminated panels mechanically stitched together form a portal frame over the doorway; full height fins span from ground to roof; bolted to ground slab; **Zone C:** 3No. adjacent full height splice-laminated panels mechanically stitched together form a vertical cantilever in the facade plane; full height fins span from ground to purlins in zone F; bolted to ground slab; **Zone D:** 5No. adjacent full height splice-laminated panels mechanically stitched together form a vertical cantilever in the facade plane; full height fins span from ground to zone E; **Zone E:** 1No. splice-laminated cold-bent panel spans as a beam between stone parapet and zone A, resist wind load applied to zone D; **Zone F:** 8No. splice-laminated cold-bent panels mechanically stitched together form a horizontal beam spanning 60° between parapets, resist wind load applied to zones A, B, C. **Zone G:** single infill panel – no structural function. A preliminary stick model of global behaviour is shown on the right.

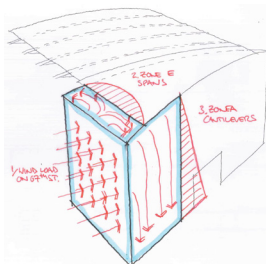


Figure 9: Structural System SS1, Wind load on 67th St. façade.

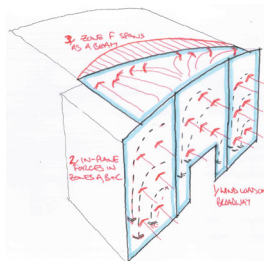


Figure 10: SS2, Wind load on Broadway façade. Note developed in-plane forces, see 3.2

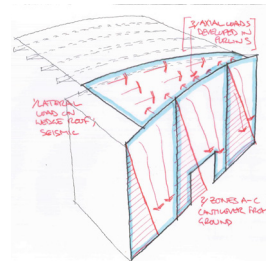


Figure 11: SS3, North/South seismic loads on the wedge are transferred into the shear walls.

3.1. Aesthetics

Original concepts considered the roof glass working as simple infill glazing panels, with loads within the roof and from the façades restrained by a grillage of steel and glass beams, similar in appearance to that used in the Apple Cube on 5th Ave. While glass is ostensibly transparent, when viewed on edge proves to be as opaque as steel – the use of glass beams in the roof was therefore not an effective method of making structure disappear to increase transparency. Rather than simply replacing steel members with glass, our aim was to remove as much structure as possible from the roof without compromising stability.

3.2. Geometry

As well as being tricky to deal with architecturally, a key structural issue is the angle between the two main façade areas. A typical perpendicular façade transfers forces in simple load paths as glazing panels simply span between fins which in turn simply span between restraining structure. Broadway runs at 60° to 67th St. which results in load paths more commonly associated with pinned truss system. This shift from perpendicular load transfers results in interaction between in- and out-of-plane forces (Figure 12): perpendicular wind loads on Broadway result in axial loads in the fin restraints (purlins) which in turn generate in-plane forces in the Broadway façade as. This geometry effect works in both directions, which leads to unintended consequences magnifying the ever present need to accommodate movements in brittle materials.

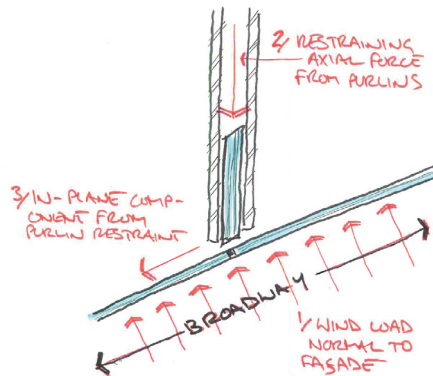


Figure 12: Typical bracing systems work assuming stiffening parallel to the direction of load. The 30 deg angle of the fins to the load direction results in in-plane force components within the façade panels.

3.3. Movements

The designer of glass structures must always be aware of strain-induced stresses, particularly in stiff connections, caused by the movement of the supporting structure. Whereas steel would locally yield plastically to accommodate movements, relax and still function in load transfer without failing, glass, as has been well documented, has no plastic zone in which to accommodate movements: sudden and complete material failure is a real risk.

In our case, the intent was to restrain the façade against the purlins which would then transfer load into the braced frame structure in the rear of the building. In the wind-load direction, this is a feasible load path. Unfortunately, the load path interaction outlined above works in both directions: forces originating in the steel purlins would push against the facade. With the steel purlins running 33m, a temperature variation of $\pm 30^{\circ}\text{K}$ would result in a thermal expansion and contraction of almost $\pm 20\text{mm}$. A typical perpendicular façade would move back and forth to accommodate this movement but in our case, with the 60° angle, this movement would generate unacceptable in-plane stresses within the façade glazing. It would also set up the situation where the main structure, concrete and steel to resisting load, is braced against the glass, a situation to be avoided unless explicitly part of the design.

3.4. The Solution

These requirements, particularly the isolation of thermal movements, geometry effects and the need for maximum transparency, were solved by isolating the wedge from the rest of the barrel vault and stone structure, consequently designing the wedge roof as a structural member – tension, shear and compression connections between panels were developed within Structural Zone F (Figure 8) to allow the roof to laterally span 20m and brace the wedge against wind and seismic loading. Similarly, Structural Zone E (Figure 8) was developed to span as a beam between the stone structure and glass.

In this way normal and lateral loads applied onto the Broadway and 67th St. facades are transferred through the roof plane, isolating structural movements and removing bracing against loads perpendicular to 67th St. Any in-plane loads in the Broadway façade are resisted by Structural Zones A, B and C which cantilever from the ground resisting overturning. See Figure 9, Figure 10 and Figure 11.

3.5. Geometry Effects Revisited

Considering the roof as a lateral beam, another effect of wedge-shape is this beam has a decreasing depth in which to accommodate bending moments and shears. A quick check identified the worst case conditions which would drive the design: Figure 13 shows the effect that a reduction in shear depth has on the local shear magnitude, with maximum shear stress at joint 7. Similarly Figure 14 shows how the location of maximum moment (therefore tension and compression forces) is shifted towards the shallower end of the section.

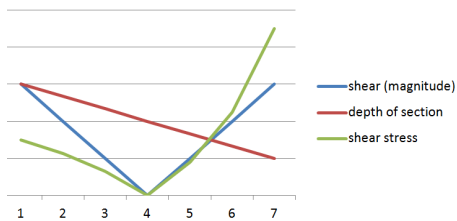


Figure 13: The magnitude of the shear in a beam varies linearly with x . With a reduction in the shear area the shear stress increases with x^2 .

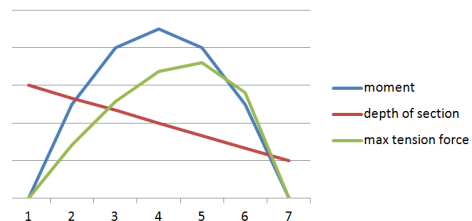


Figure 14: the moment in a beam varies with x^2 . With a reduction in a section depth, the maximum tension and compression forces is shifted towards the shallower section.

3.6. Pushing the envelope

The architectural intent was to have a curved roof to follow the IGU roof behind which posed a problem. The size of the largest wedge roof panels (12m x 2.3m) would not allow for chemical tempering and splice laminating would not work for hot-bent heat-treated glass due to inconsistent curvatures and misalignment. The solution to this problem allowed the realisation of a fourth requirement: the conscious effort to push the envelope with each step in structural glass use. The panels were cold-bent and splice-laminated in a mould which forced a curvature into the individual plies. The result is a heat tempered curved jumbo laminated panel which would not be possible under any other fabrication method known at this time.

4. Modelling of Glass Structures

Once the design concept is clear, medium scale structures such as the Apple Store UWS are modelled in FE analysis programs to verify the intended load paths and to determine the magnitude of forces through connections. The global model is built up from simple stick elements (Figure 8, right) and, once the expected structural behaviour is confirmed, the type and number of mechanical connections can be determined and local models of the connections constructed. The local models advise the engineer of connection stiffness which feed back into the global model – the change from idealised connections can affect the load path which in turn may vary the force transferred through a fitting. Where all fittings are to be visually identical, the worst-case defines the overall appearance.

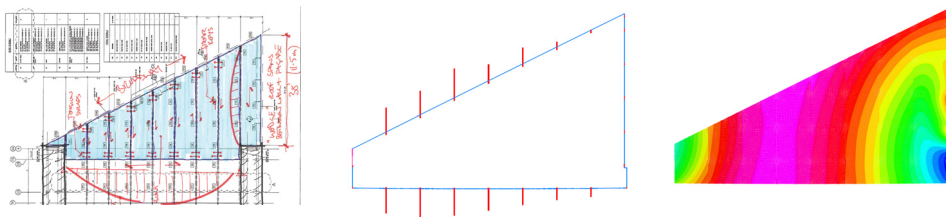


Figure 15: From left to right: structural scheme of horizontal beam in the roof plane; tension in fittings on top and bottom of the “beam”; deflections of the “beam”. Forces in the fittings are isolated and applied to more detailed models (below). The fittings in the wedge transfer combinations of tension, compression and shear.

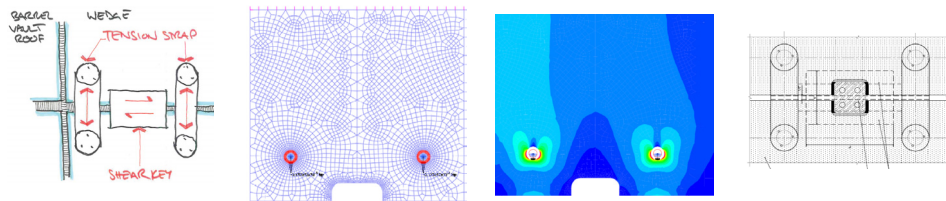


Figure 16: Forces from the global model are extracted, connection concepts are developed and applied to detailed models showing local stress concentrations which dictate final connection design. The example above shows a combination of shear and tension forces to be transferred on the bottom edge of the wedge.

5. Glass Design Stresses

Design of glass structures in the US is governed by the ASTM E1300 and is the basis for the design of Apple UWS. We will follow the example of a single 10mm sheet of FT glass which will outline eventual worst-case design stresses once all design factors have been considered.

5.1. Nominal thickness, body and edge stress

The first step is to convert the 10mm nominal thickness to 9mm, the minimum actual thickness which can be assumed in calculations. ASTM E1300 also describes allowable glass stresses with values given for glass surface and edge stresses. Considering our 10mm sheet of FT, E1300 describes an allowable surface stress of 93.1MPa, which reduces to approx. 80% capacity (73 MPa) when considering edge stresses.

5.2. Load duration

A key factor in the design of structural glass is the effect load duration has on failure probability: sub critical cracks grow under a sustained load, which in turn grow to form critical cracks cause material failure. To mitigate this effect, a Load Duration Factor, LDF, is calculated to further augment allowable stresses.

E1300 allowable stresses are based on 3s loads: each load is multiplied by the relevant LDF which allows the combination of long term loads (self-weight) and medium term (live access/post-failure) with short term (wind) 3s loads. The load duration factors for AN glass are tabulated in E1300 (Table X6.1) – these can be extrapolated to HS and FT glass by reference to Equation 1 (or X7.1 in E1300):

TABLE X6.1 Load Duration Factors

NOTE—Calculated to 8/1000 lites probability of breakage (see 3.2.9).

Duration	Annealed
3 s	1.00
10 s	0.93
1 min	0.83
10 min	0.72
60 min	0.64
12 h	0.55
24 h	0.53
1 week	0.47
1 month	0.43
1 year	0.36
beyond 1 year	0.31

TABLE 1 Glass Type Factors (GTF) for a Single Lite of Monolithic or Laminated Glass

Glass Type	GTF	
	Short Duration Load	Long Duration Load
AN	1.0	0.5
HS	2.0	1.3
FT	4.0	3.0

$$q_3 = \sum_{i=1}^{i=j} q_i \left[\frac{d_i}{3} \right]^{\frac{1}{n}} \quad (1)$$

Where q_3 = the magnitude of the 3s duration uniform load, q_i = the magnitude of the load having duration d_i and $\left[\frac{d_i}{3} \right]^{\frac{1}{n}}$ the load duration factor.

In this way long term loads, for example, are increased by approx. 80% when applied to Heat Strengthened glass and approx. 50% when applied to Fully Tempered glass. Putting these together, if our 10mm FT sheet was under permanent dead load, our design stress would reduce to 49 MPa on an edge.

5.3. Failure probability

The values above are based on a failure probability (P_{failure}) of 8 in 1000 samples. For key structural members and overhead glazing, where consequence of failure is considered important for safety, the values are adjusted to allow for a $P_{\text{failure}} = 1:1000$. As a shorthand, allowable design stresses for key members are approx. 25% lower than those of non-critical elements. Therefore our 10mm FT sheet, when used as a structural member, has its allowable stress reduced to 36 MPa.

5.4. Edge bearing

As well as considering stresses on glass edges and surfaces, experiments with fitting design, carried out by Seele GmbH and Sedak GmbH, have shown that failure stresses caused by direct bearing on hole edges are reduced by up to 66% compared to secondary edge stress due to bending. If our 10mm FT panel were to have a hole through which permanent dead load was transferred directly, we would limit design to 25MPa.

5.5. Cold bending

The several 10mm plies are cold bent and laminate-spliced to form the roof of Apple UWS, a process which further reduces the capacity of the glass. Falling back on a simple calculation, simple locked-in stresses can be calculated:

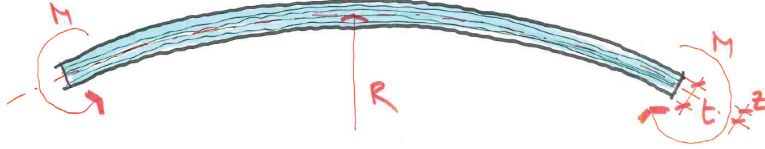


Figure 17 Cold bending by inducing a moment on panel edges, results in locked-in stresses

$$\frac{\sigma}{y} = \frac{M}{I} = \frac{E}{R} \quad (2)$$

Where σ = stress, M = moment, E = Young's Modulus, y = distance from neutral axis ($= t/2$), $I = 2^{\text{nd}}$ moment of area, and R = radius

Forcing a 50m radius cold-bend into the glass (assuming no holes or other areas of stress concentration) results in an additional locked-in stresses (both tension and compression):

$$\sigma = \frac{E}{R} \cdot y = \frac{70\text{GPa}}{50\text{m}} \cdot 5\text{mm} = 7\text{MPa} \quad (3)$$

The final design stress for a cold-bent 10mm structural FT glass, which has long term load applied directly to a hole-edge, such as with a bolt, has decreased to 18MPa, only $24/93 = 26\%$ of its full capacity.

Of course all these stress effects do usually not occur at the same time, and are only given as a guide to illustrate what the Structural Engineer must be aware of to successfully design in Structural Glass. It is in no way an exhaustive list, post failure for example has not been covered, with each structure needing to be designed on its own merits to local conditions and requirements.

6. Conclusions

Using the most recent Apple Store, Upper West Side, New York, NY, as a case study example, this paper outlines the work carried out to design large glass structures, introducing structural principles by which maximum transparency is attained, showing how these principles drive design, how the design is verified with calculations and finite element analysis and charting the variables the Structural Engineer must take into account when transferring load through glass.

It is hoped that boundaries have been pushed in the use of structural glass, that Engineers working in the field of have seen how the state of the art has progressed, that

Challenging Glass 2

Engineers not familiar with structural glass design have gained an insight into the unique design requirements, and Architects and Designer has gained a deeper understanding of the challenges facing the Structural Engineer in realising visions of transparent structures.

7. Acknowledgements

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